PHYSICAL MODELLING AND DESIGN ASPECTS FOR A NEW MEGA-YACHT MARINA

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Extensive preliminary studies and a large-scale 3D physical model study were conducted to support the design of a proposed mega-yacht marina project at Bridgetown, Barbados. A faithful reproduction of the marina, the surrounding bathymetry, and the adjacent shoreline was constructed and used to gauge the performance of the marina in a range of operational and extreme seastates, and to enhance and verify the preliminary designs. The physical model was used to investigate many key issues including: wave agitation within the marina, wave overtopping, wave impact forces, the stability of revetments and scour protection stone, and the behaviour of moored yachts. Notable outcomes of the study included the optimal length and crest elevation along the curved breakwater, the strategic positioning of two piers and a revetment within the marina basin, and the determination of suitable design loads for various breakwater components.

Keywords: Coastal Engineering, Physical Modelling, Mega-Yacht Marina, Wave Agitation, Wave Impact Forces

INTRODUCTION

The Government of Barbados, through Barbados Tourism Investment Inc., is in the process of promoting the regeneration of the waterfront at the north end of Carlisle Bay near Bridgetown, Barbados. A central feature of this development is a planned marina constructed at the Pierhead, an existing headland at the north end of the bay. The marina is intended to anchor and enhance the value of subsequent land-based improvements and is fundamental to the larger development goals. However, the marina design must be economically and operationally viable, thus necessitating a rigorous planning and design process.

Extensive planning studies were conducted as part of the design of the marina and surrounding areas. Activities included a thorough investigation of existing conditions, identifying the marina function and layout, and determining the physical processes that would define the design conditions for the facility. In particular, a market study was conducted to assess the demand for such a facility located in Barbados, as well as desired characteristics and services for the marina.

Preliminary design activities focused on refining the project definition and conducting more detailed design studies to further develop marina concepts. Notable investigations included site data collection (geotechnical studies, wind, wave, and current measurements, etc.), numerical modelling to determine environmental design conditions, evaluation of numerous design configurations (dockage and vessel mooring layouts), and preliminary structural design (definition of loads, evaluation of structural alternatives, and seismic considerations) as well as utility routing.

Once the preferred concept layout was identified, the preliminary design was verified using a large-scale physical hydraulic model, carried out at the National Research Council of Canada (NRC). The model was used to assess and resolve a wide range of issues, and to optimize the marina design to maximum performance versus cost.

The main structure tested in the model was comprised of a curved vertical-wall breakwater (sheet pile construction) with structural backfill, and was capped with concrete and architectural features at various elevations to provide both public and private promenades. Other elements were also tested, including rubble-mound revetments (used for shoreline protection and wave attenuation around the marina), protective measures to resist scour at the main structure, and also verification of the effects of the proposed marina on nearby existing facilities.

TEST FACILITY AND MODEL CONSTRUCTION

The selection of a suitable geometric scale is a key element in the design of any physical model. Although many factors are considered, the decision hinges on striking a compromise between two important conflicting requirements: minimizing scale effects (particularly with respect to differences in Reynolds number) by selecting as large a model scale as possible, and minimizing boundary effects by modelling as large an area as possible (which requires a small model scale). The dimensions of available facilities are the capability of critical equipment, including wave machines, must also be

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taken into consideration. For this study, testing was carried out at an undistorted geometric scale of 1:50 in NRC's 30 x 36 metre Multidirectional Wave Basin (MWB), sketched in Figure 1. The MWB is equipped with a 30 metre long 60-segment directional wave generator capable of generating short crested irregular waves approaching from a ~60° range of directions.

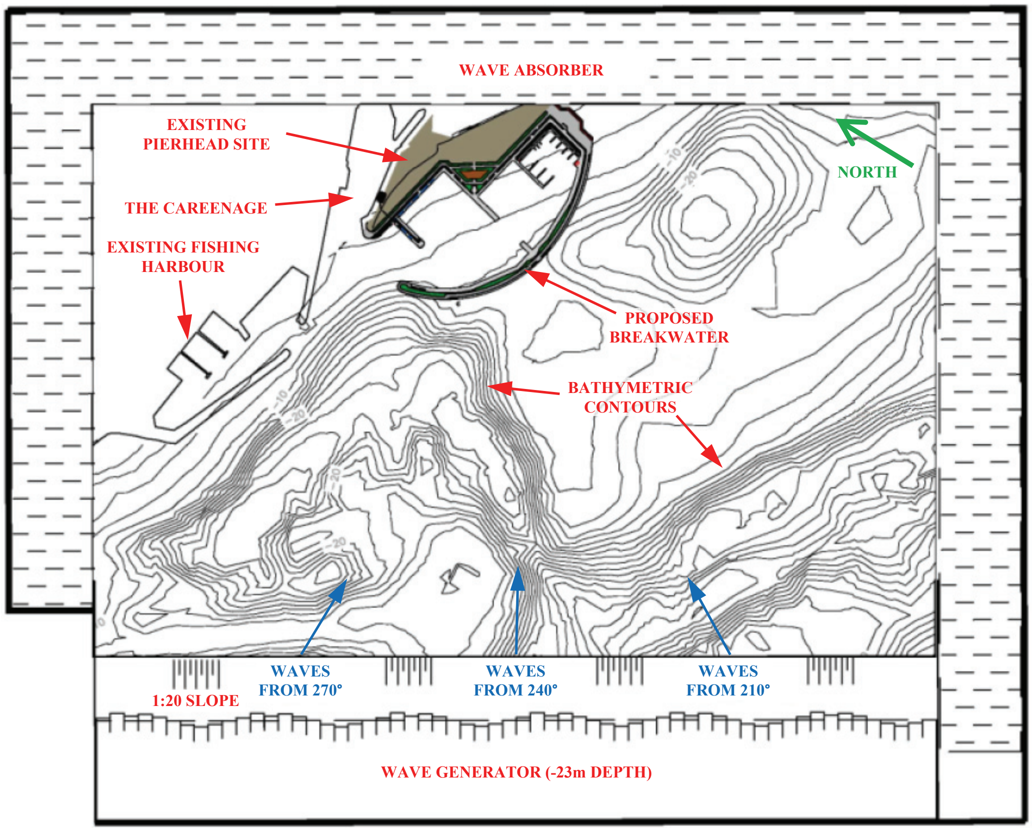

Figure 1. General layout of the 1:50 scale physical model in the 30 x 36 metre MWB facility.

Typical wave conditions at the project site include a range of seas approaching from SSW and swells approaching from NW. On rare occasions, the site is also subject to larger waves generated by hurricanes and tropical depressions passing near Barbados. Statistical and numerical analysis performed as part of the preliminary design determined the effective water levels, wave heights, and wave periods to be used in designing the new marina and in the physical model study.

The physical model layout and the model bathymetry was designed and constructed so that the performance of the mega-yacht marina could be assessed in waves approaching from a wide range of directions. The model was oriented within the basin so that the segmented wave generator lay along the 150°-330° bearing. With this orientation and layout (shown in Figure 1), the marina could be tested in short-crested waves approaching from a range of mean wave directions between ~210° and ~270°. Significant wave heights in the model ranged from 0.5 to 10 metres in height (offshore), with peak wave periods ranging from 7 to 16 seconds.

Wave conditions near the marina are strongly influenced by the local bathymetry, which is highly irregular and includes several offshore reefs and a deep channel that leads into the Careenage area. These features play a critical role in influencing how the offshore waves transform as they approach the coastline. It was therefore important to replicate the existing bathymetry at the site in the physical model as closely as possible. Working from bathymetric surveys conducted in 2010-11, an accurate reproduction of the local bathymetry was constructed down to the -20 metre depth contour. The bathymetry was constructed using a network of fibreboard templates, backfilled with fine gravel, and capped with a skin of concrete that was screeded to match the elevations defined by the templates. The resulting bathymetry was generally accurate to within ±3 millimetres, equivalent to ±15 centimetres at full scale.

The nearshore bathymetry in the vicinity of the proposed marina was initially configured to represent existing conditions (without the marina). Accurate scale models of the existing neighbouring coastal structures were then constructed in the physical model. These neighbouring structures included the existing rubble-mound structure at the Pierhead site, the nearby fishing harbor (featuring internal stone revetments, a southern rubble-mound breakwater, and a north Dolos-armoured breakwater), and the Careenage entrance with dry-dock and pile-supported boardwalk. The model was outfitted with twenty wave gauges and tests were conducted to tune and calibrate the incident wave conditions generated along the offshore boundary of the model to match specifications derived from numerical modelling, and to determine the properties of the incident wave conditions near the project site. Following these wave calibrations, the bathymetry was modified to replicate future conditions, including dredging for the entrance channel and marina basin.

Next, an accurate scaled replica of the proposed mega-yacht marina was constructed in the model. The breakwater was modelled as an assembly of over twenty-five wooden segments (see Figure 2). Each segment was designed with the ability to easily modify the crest elevation in order to investigate the impact of overall structure elevation on wave overtopping and wave agitation within the marina. In the model, a total station was used to guide and control layout of the new breakwater and marina to match coordinates from CADD drawings. Vertical control was maintained throughout the construction process using precision survey equipment. At the time of construction, the crest elevations of the wooden segments used to construct the model breakwater were installed to within a ±1 millimetre vertical accuracy, corresponding to ±5 centimetres at full scale.

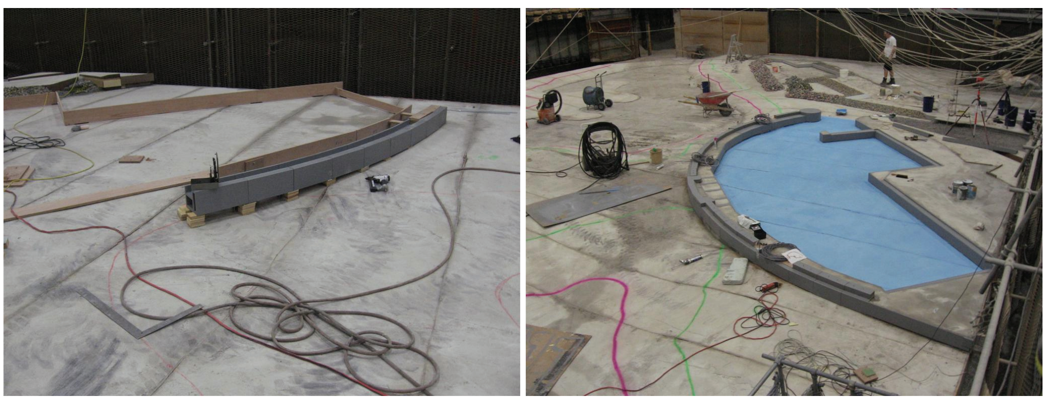

Figure 2. Views of the model during construction of the new marina.

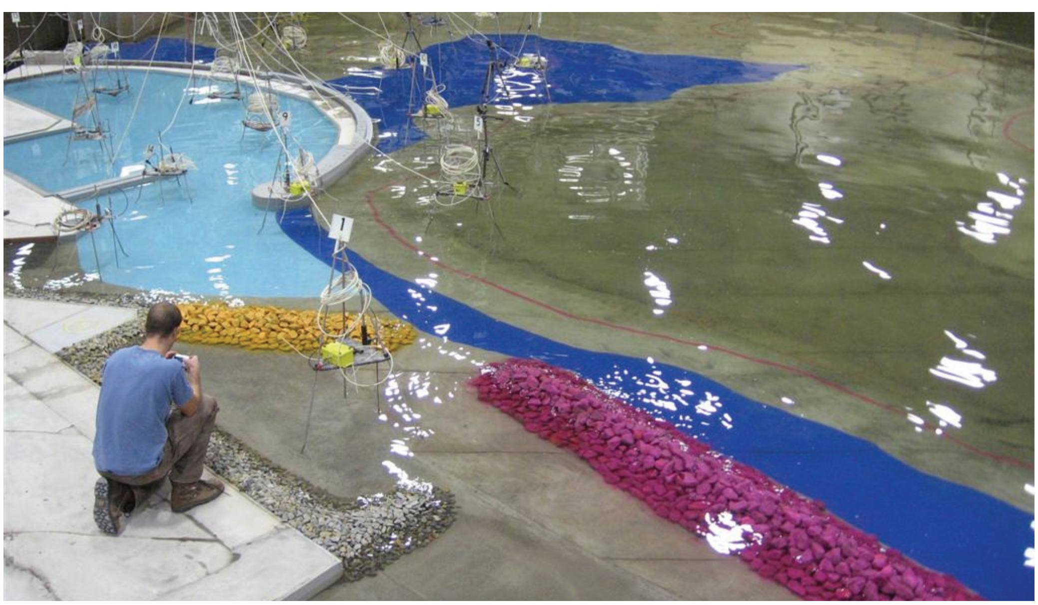

Figure 3. Physical model of the proposed mega-yacht marina.

The interior layout of the marina was modelled to match the most recent dockage layout plan available at the time of model construction. Several interior marina features were modified or added during subsequent testing. The model bathymetry was painted to highlight certain key elements and depth contours: light blue to represent the -7 metre dredge area inside the marina, dark blue to represent depths between -7 and -10 metres, while a purple line was painted to indicate the -15 metre depth contour. A photo of the completed marina model during testing is shown in Figure 3.

INSTRUMENTATION

Twenty capacitance-type wave gauges were deployed to measure wave conditions at specific locations in the model domain. Four gauges were located approximately 500 metres offshore from the edge of the proposed marina footprint. During the wave calibration process, the command signals used to drive the wave generator were adjusted so that the wave conditions measured at these four probes were in agreement with target incident wave conditions derived from numerical modelling. The other sixteen gauges were distributed in various locations both inside and surrounding the marina.

Three simple, accurate, and reliable overtopping measurement systems were developed for use in the model. Each system consisted of a water storage reservoir, a capacitance wave gauge to measure the depth of water in the reservoir, and a metal tray to convey the overtopping flow from the crest of the breakwater into the reservoir. The systems were deployed at five distinct locations within the marina to determine overtopping flows.

Sixteen pressure sensors were installed in the sea-side vertical wall of the model breakwater to measure wave-inducted hydrodynamic pressures and establish loads for use in design. Fourteen of the sensors were arranged in three arrays that could be placed vertically or horizontally to develop wave pressure profiles. Later in the testing program, the three pressure sensor arrays were grouped closely together and relocated between tests to measure wave impact loads at seven distinct locations along the breakwater.

Four remotely-operated digital cameras were used to monitor the movement of various rock structures, including the scour protection and revetments. The four cameras were mounted on stationary tripods and aimed to view critical areas. Since each camera remained fixed throughout a test series, the movement of individual stones could be detected by comparing photographs taken at different times. In addition, four video cameras (with remote pan, tilt, and zoom capabilities) were installed above and beside the model marina and used to digitally record all tests.

WAVE AGITATION WITHIN THE MARINA

One of the primary study objectives was to examine the various factors influencing wave agitation within the marina, shown in Figure 4. These influences include the main breakwater length (position of breakwater head), the breakwater crest elevation, alternative entrance configurations, the arrangement of revetments within the marina, and the configuration of piers within the marina. For typical operational wave conditions, wave disturbance within the marina was the result of wave energy penetrating through the marina entrance; however, for more energetic wave conditions (and higher water levels), water passing over the breakwater crest became an increasingly important factor.

Wave Agitation Criteria for the Marina

In collaboration with stakeholders, wave agitation criteria for the marina were developed based on reference standards published by ASCE and PIANC, as well as British and Spanish Standards. None of these standards specifically addressed criteria for mega-yachts. Therefore, the target agitation criterion was a hybrid based upon published values relating to both recreational yachts and small cargo ships. The target criteria were defined as follows:

- To provide safe and comfortable boat-related operations, wave conditions are not to exceed H_s = 0.15 metres for more than 15% of the time.
- To minimize damage to boats moored in the marina, wave conditions are not to exceed H_s = 0.30 metres for a 1-year event (excluding named tropical storms and hurricanes).
- To minimize damage to floating docks and other infrastructure in the marina, wave conditions are not to exceed H_s = 0.75 metres during the 10-year event. During such an event (e.g. a nearby hurricane) there should not be vessels in the marina.

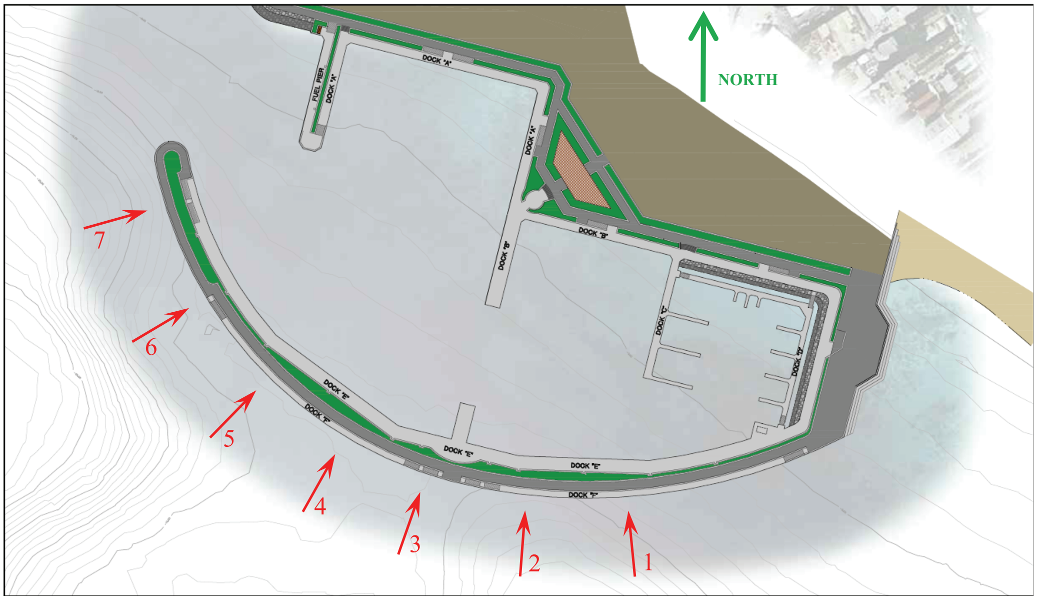

Figure 4. Site plan for the proposed mega-yacht marina.

Effect of Breakwater Length on Wave Agitation

The preliminary design of the mega-yacht marina that was constructed in the physical model featured an arc-shaped breakwater that was approximately 540 metres long. Several shorter breakwaters were also tested in which the breakwater roundhead was shortened by 20, 40, and 50 metres, thereby increasing the width of the marina entrance.

Figure 5 shows the relationship between breakwater length and wave agitation for various wave conditions from four different directions (210°, 225°, 240° and 270°). In each case, the breakwater crest height was set at +3.5 metres. Due to positioning of the breakwater entrance relative to the incident waves, the influence of breakwater length of marina agitation was smaller for wave approaching from 210° that for other directions. However, waves approaching from 270° were able to penetrate more easily into the marina when the entrance was enlarged.

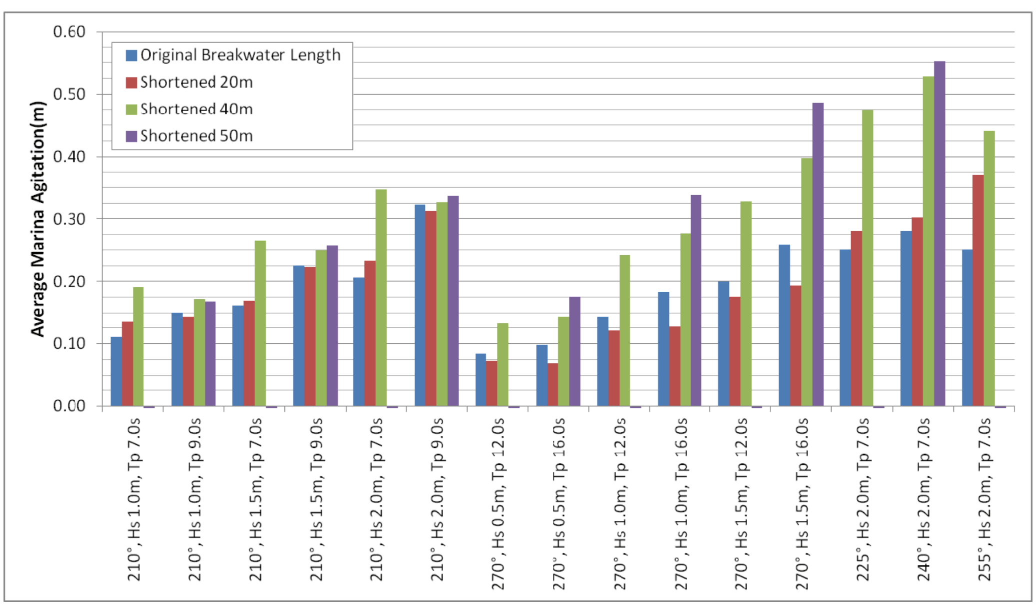

Figure 5. Effect of breakwater length on average marina agitation.

The addition of a revetment at the marina entrance was found to be effective at reducing wave heights in the marina (during day-to-day operational conditions). In addition, interior revetments located along the east wall help to reduce wave heights at the eastern part of the marina basin where floating docks and smaller vessels may require greater protection.

With consideration for other elements (namely the entrance revetment), it was concluded that the length of the breakwater could be reduced by 40-50 metres from its original length and still provide an acceptable level of protection for the marina. This change alone represents a substantial cost savings for the project.

Effect of Interior Revetments on Wave Agitation

With vertical walls surrounding most of the marina, wave energy in the basin will build unless there is some way to dissipate this energy. Numerical modelling had suggested that a revetment along the east wall would be effective in reducing the overall marina wave agitation. Therefore, the effect of adding a revetment along the east end wall of the basin was tested in comparison to having all vertical walls (see Figure 6). Moreover, the relative effect of extending this same revetment to cover a portion of the north wall was also investigated. The easterly position of the revetment was chosen as a result of the wave focusing at the end of the marina, in addition to the desire to minimize the wave heights near the area which may contain floating docks and smaller vessels at the east end of the marina.

The general conclusions from these tests were that the revetment was most effective in reducing the wave height for the shorter period waves, while the longer period swells (e.g. 16 seconds) saw less reduction in wave height. The revetment became less effective during larger storm events, when wave overtopping occurs more frequently and more intensely, and wave energy is spilled over the marina walls around much of the perimeter. Both of these revetment features are recommended for the final design.

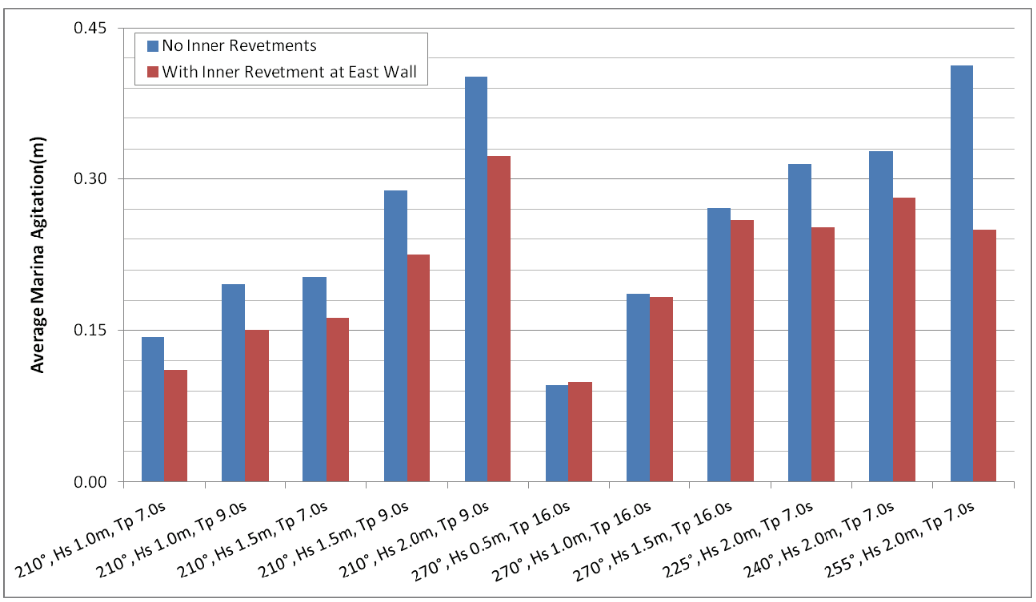

Figure 6. Effect of inner revetment at east wall on average marina agitation.

Effect of Marina Pier Configurations on Wave Agitation

The marina conceptual layout featured two large interior piers (Piers B and C) for mooring vessels (see Figure 4). These piers were tested as pile supported structures and also as solid structures that would provide some wave protection (see Figure 7). In general, closed piers prevented some wave energy from penetrating into the inner (eastern) portion of the marina; however waves that did penetrate into this area tended to be trapped in this area despite the revetments. The presence of closed piers also tended to slightly increase the wave agitation in the outer (western) part of the marina. Based on the results of the testing, it was determined that making Pier C solid was of limited value since it did little to reduce wave heights. Constructing Pier B with a solid face effectively reduced the wave conditions in the inner basin while only marginally impacting the waves in the outer basin.

Initial numerical modelling, along with visual observations during model testing suggested that there was some focusing of wave energy along the inside of the outer breakwater due to its curved shape. A number of tests were completed where a small stub groyne was installed opposite Pier B, on the inside of the main breakwater. Addition of this stub groyne led to a marginal improvement in agitation levels for the inner portion of the marina.

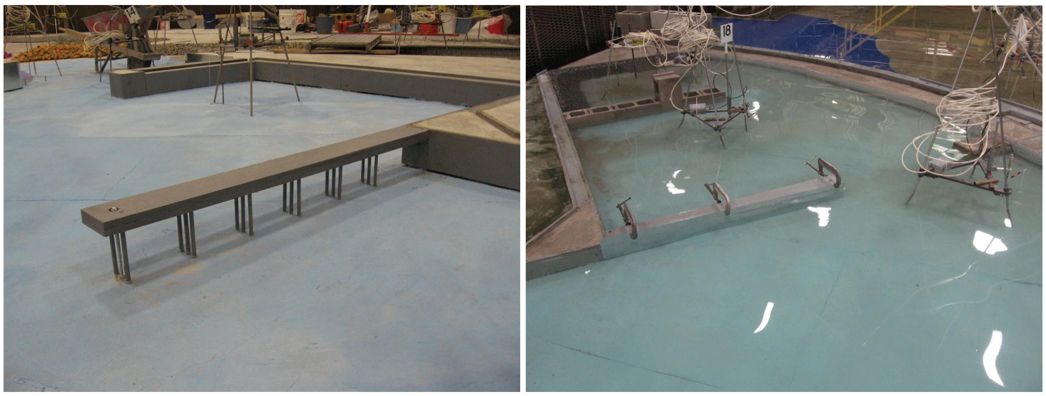

Figure 7. Various marina pier configurations were investigated in this study.

WAVE OVERTOPPING

The model breakwater was constructed in such a way that allowed the crest elevation to be varied quickly yet precisely. The outer fair weather dock was built to a fixed height of +2.0 metres, while the inner dock was fixed at +1.5 metres. The central public walkway was adjustable from +2.0 to +5.0 metres elevation (see Figure 8). Three continuous breakwater crest heights were initially tested in the model (over the full length of the structure), while additional tests examined local variations in breakwater crest height. Modifications to the crest of the breakwater, in the form of strategically placed walls and other structural features, were investigated and found to be effective at reducing the amount of overtopping flow reaching and passing over the marina-side of the structure.

Initial observations of the wave overtopping indicated that the complex bathymetric features in front of the structure caused the wave overtopping to vary considerably along the length of the main breakwater.

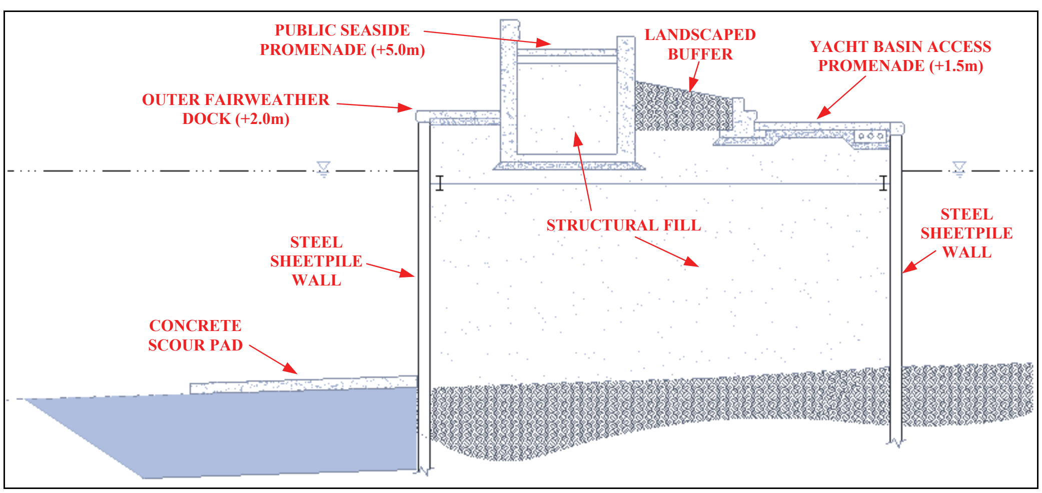

Figure 8. Typical cross-section through the main breakwater.

Effect of Breakwater Crest Elevation on Wave Agitation

During mild overtopping events, most of the overtopping flows passing over the crest would land on the inner dock area and spill relatively gently off the dock into the marina. During more severe overtopping events, green water flows would pass directly over the breakwater crest and land on either in the inner dock area or directly onto the waters within the marina, causing aggressive agitation. The agitation in the marina due to any overtopping flows can be described as very short period waves propagating in multiple random directions.

Figure 9 shows the effect of breakwater crest elevation on marina agitation for various wave conditions approaching from four different directions. It is apparent that, for the smaller storm events when overtopping is very mild, a lower-crested structure performs nearly as well as a higher-crested structure. However, for the 10-year and larger storm events, the degree of wave agitation increases strongly with reduced crest elevation.

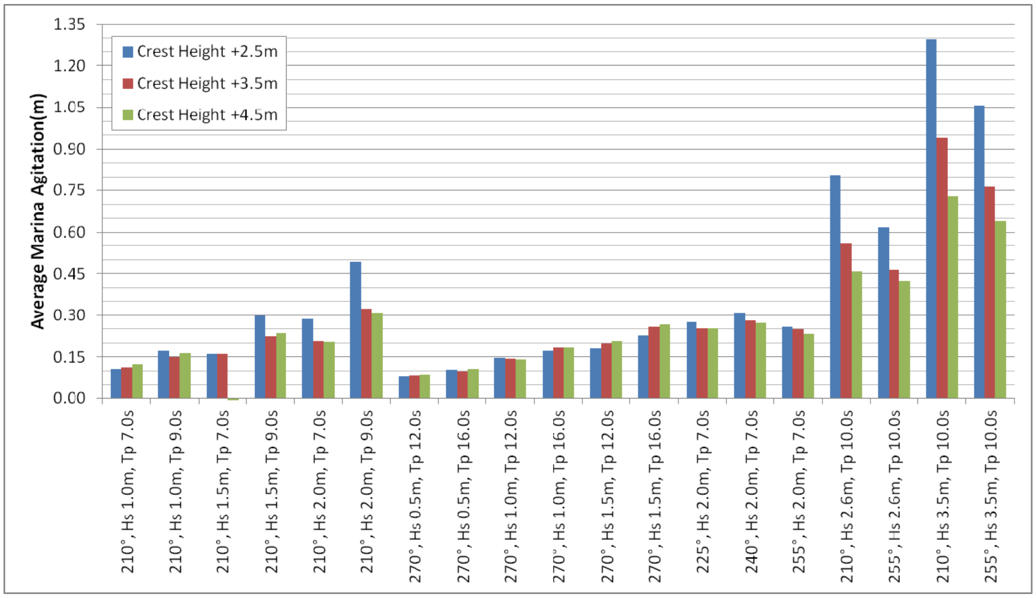

Figure 9. Effect of crest height on average marina agitation.

Effect of Breakwater Crest Elevation on Wave Overtopping

The criteria for wave overtopping relate to two processes: wave agitation inside the marina, and damage to infrastructure and amenities on top of the breakwater. Thresholds for wave overtopping limits were based on guidance published by Eurotop (2007), which suggests particular mean discharge thresholds for damage behind a seawall:

For property behind a sea defence:

- 1 L/s/m Building structure elements
- 10 L/s/m Sinking small boats close to walls, damage to larger boats
- 50 L/s/m Significant damage or sinking of larger yachts

For promenade or revetment sea walls:

- 50 L/s/m Damage to grassed or lightly protected promenade or reclamation cover
- 200 L/s/m Damage to paved or armoured promenade behind seawall

Visual observations (both real-time and from review of video) showed that some regions along the breakwater crest were more prone to overtopping than others as a result of wave direction, nearshore bathymetry, and local water depth at the structure toe. The region most prone to overtopping during moderate conditions was found to be the central part of the structure, caused by the higher wave heights in this region due to wave refraction and focusing, as well as the shallower water that resulted in steeper waves and more breaking.

The variation of mean (time-averaged) overtopping flowrate (near the central part of the structure) with breakwater crest height is presented in Figure 10 for several different wave conditions. For all wave conditions and water levels, up to and including the 20-year wave event, recorded overtopping rates were generally below the 50 L/s/m threshold for all crest heights examined. For the 100-year return period events, mean overtopping flowrates were more severe and exceeded the acceptable thresholds.

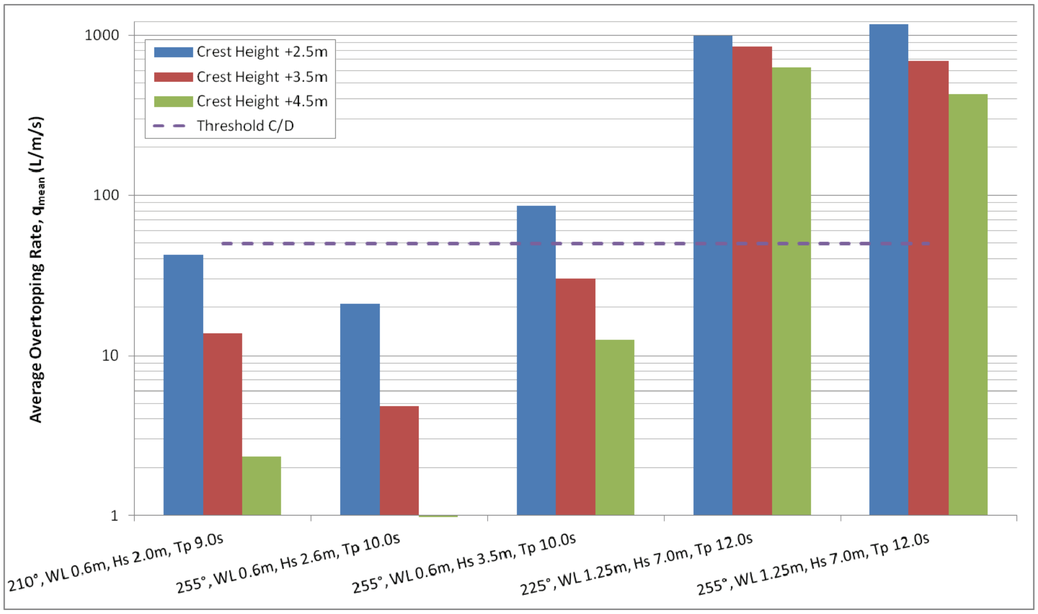

Figure 10. Influence of breakwater crest elevation on mean overtopping discharge.

It was determined that the crest elevation of the breakwater should be in the range of +4.0 metres, with local increases and decreases in areas with greater or lesser levels of risk. The elevations were established by considering both wave agitation inside the marina, and potential damage to infrastructure due to the flow of overtopping water during significant storm events.

WAVE IMPACT PRESSURES

Measurement Method

Wave induced pressure fluctuations were recorded on the seaward vertical face of the breakwater throughout the duration of the study. Two key challenges in dealing with wave pressures are in understanding the spatial and temporal variability of the pressure fluctuations and related forces. In a temporal sense, waves can exert very brief pressure impulses as well as longer duration quasi-steady pressures (see Figure 11). The longer duration pressures are lower in magnitude, but can have greater influence on a caisson structure since they can last over ~1-3 seconds (depending on the wave period). The impact pressures can be considerably larger, but tend to prevail over very short durations. Considerable spatial variation in peak pressures was observed in tests with the horizontal pressure sensor arrays located near the water line; in some tests, one sensor would show a large impact pressure pulse, while the two sensors on either side did not show the same impact pressure pulses. In general, the impulsive pressure peaks tend to exhibit considerable spatial variability, whereas the quasi-static pressures tend to be more spatially consistent and homogeneous.

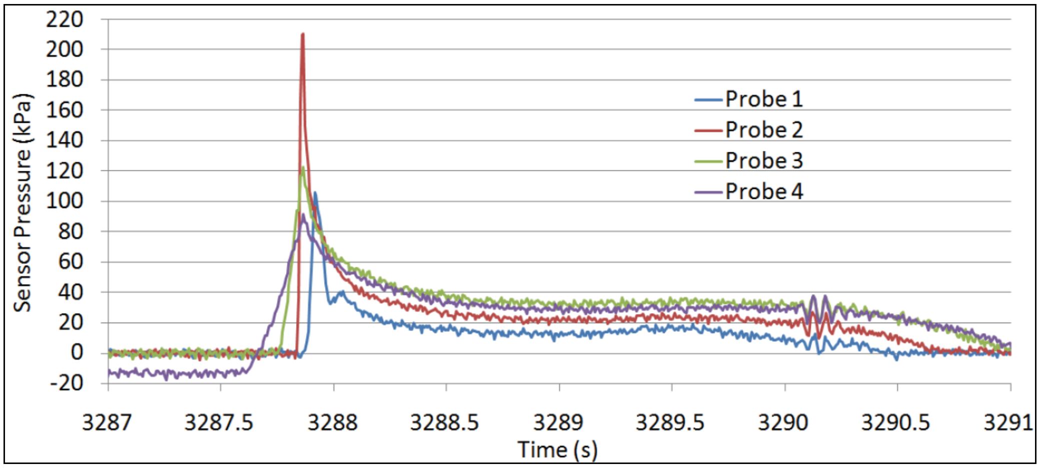

Figure 11. Example of pressure fluctuations measured during a single wave impact.

It became evident that meaningful wave pressure data would need to be obtained over a region of the breakwater, in order to develop a regional pressure distribution that could be converted into a global load that could affect the stability of the structure. To accomplish this, three vertical pressure sensor arrays were installed in proximity to each other and the measured pressures were integrated to estimate the horizontal force acting on a 20 metre long breakwater section (see Figure 12). In addition to the spatial averaging described above, temporal smoothing of the estimated global load time histories was also used to further reduce the influence of the short duration pressure impulses, and develop load estimates for use in design.

Figure 12. Array of pressure sensors on the seaward face of the model breakwater.

Measurement of Wave Impact Forces

Wave pressure measurements were carried out for a range of test conditions, with the greatest emphasis being on the extreme waves associated with the 100 and 500-year events (see Figure 13). During each test, the 10 largest pressure events were averaged to provide a more statistically stable value for making comparisons of pressures along different sections on the breakwater. Pressure measurements were obtained at seven locations along the breakwater (refer to Figure 4). Analysis of the measurements showed increasing wave forces moving from location 1 (east) to 6 (west), with the peak loads at location 7 being slightly lower than those at location 6. A notable observation from the data was the ratio between the single largest impact and the average of the top 10 events. The test results indicate lower variability in peak pressures towards the east end of the breakwater, and greater variability towards the western end of the structure.

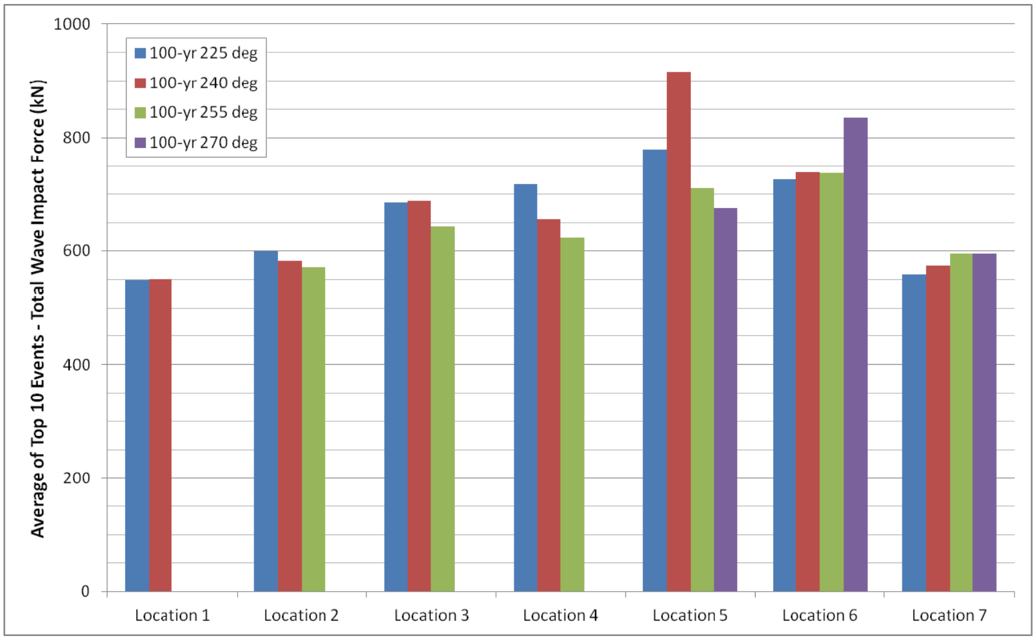

Figure 13. Average of top 10 wave impact forces for 100-year events.

STONE STABILITY

Armour stone is to be used for scour protection along the seaward face of the breakwater, and for armouring revetments at the marina entrance and within the marina basin. General performance criteria suggested that the armour stone to be used for each particular function should be appropriately sized for stability during design conditions; however, during particularly large storm events, some level of damage could be tolerated, since there is typically ample time between design events to repair small sections of scour stone or revetment. Suitable locally available stone was identified for the scour protection as well as the revetments at the marina entrance and within the marina.

Scour Stone

Various sizes of scour stone were tested around the outside of the breakwater and around the head of the structure. Scour stone was placed directly on the concrete model bathymetry, which is a conservative approach since the stone (a) is more likely to slide and/or roll on a smooth concrete bottom, compared to on natural bottom conditions, and (b) was most likely placed higher in the water column than reality since the prototype installation may see the top of the stone apron placed flush with the elevation of the surrounding existing bathymetry in some regions.

The performance of the scour protection was visually assessed and supported by the use of four fixed digital cameras, positioned on tripods to view different sections of the breakwater. After placing the scour protection, the stones were painted with different colours to delineate the different sizes/classes of stone and also to facilitate the detection of stone movement. Still photos were collected before and after each wave condition for later analysis to determine if and when any damage occurred (see Figure 14).

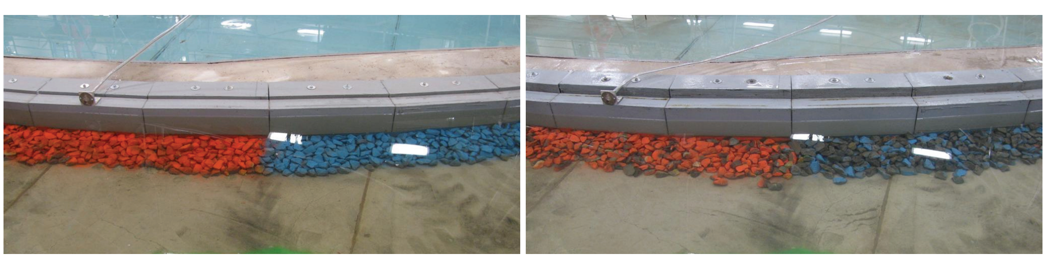

Figure 14. Scour protection before and after exposure to a 100-year event.

The one region that was particularly susceptible to damage was the eastern portion of the breakwater, where large waves were travelling parallel to the structure and wave breaking was occurring in these shallow regions. The wave breaking process pushed the scour stone in an easterly direction. In reality, these stones would have settled into the underlying sand layers (in prototype) and would not have moved as excessively as was observed in the physical model. This instability may be alleviated by partially excavating the stone into the seabed in these shallower regions, which is also beneficial for navigation and mooring on the outer fair weather dock.

Entrance Revetment

The agitation analysis revealed that a revetment located at the marina entrance (just to the west of fuel pier) provided a valuable means of reducing wave energy. Installing this revetment reduced the wave heights in the marina by the equivalent of adding approximately 10 metres to the breakwater length (therefore allowing a shorter breakwater). Several configurations of this revetment were tested as part of the physical model to determine the optimum configuration in terms of performance and stability (see Figure 15). Due to its location, it was known that the revetment would be exposed to larger waves during design conditions and would therefore require larger armour stone.

Figure 15. Various entrance revetment configurations were modelled and assessed.

Stability testing showed that the selected armour stone was stable under all conditions leading up to the 100-year storm event. During the 100-year event, minor damage was sustained (i.e. in-place movement of individual units) above the waterline. However, no individual stone units were shown to have moved off the revetment slope (and into the marina navigation channel or the Careenage), even under repeated exposure to the 500-year storm.

Interior Revetments

The revetments at the east end of the marina are generally exposed to very small wave conditions since they are so well protected. The largest amount of damage to these revetments in the model was observed due to the following mechanisms:

- Overtopping of the main breakwater: Large volumes of water pass over the main breakwater and the flow of this water onto the south end of the marina revetment resulted in considerable displacement of stone.
- Surging over walkways: During extreme wave conditions and extreme water levels there is a large volume of water that moves onto and off the walkway. When this water flowed back off the walkway (+1.5 m elevation) onto the revetment, it caused washing of stones off the crest of the revetment, and down the revetment slope.

Due to these processes, it may be necessary to increase the size of the armour stone close to the main breakwater, and also use a larger armour stone near the crest. Setting a crest elevation close to the elevation of the walkway will also be beneficial, and a crest width of three stones will help reduce the pulling of stones down the slope.

MOORED VESSEL MOTION

Although it was not a primary objective of the physical modelling study, a qualitative simulation and assessment of moored ship motions was also conducted. Three model yachts were constructed for use in the physical model, representing vessels of 20, 40, and 70 metres in length. The vessel shapes were chosen based on what were considered to be typical vessel types that may be moored in the marina. Wave heights were measured extensively throughout the basin and were the primary source of data use to assess wave agitation. The model yachts were intended to provide a qualitative assessment of the severity of motions that could be expected, as well as the different modes of motion. Yacht motions were assessed through visual observations supported by analysis of video recordings.

The model yachts were constructed from high density tooling foam, which was hollowed out to remove weight and permit appropriate ballasting. Lead weights were added until the correct water line was achieved for each yacht. While a formal process of matching the periods of oscillation in pitch and roll was not completed, a roll comparison was performed. The roll periods of the vessels were adjusted by lowering or raising the ballast, or moving the ballast weights laterally, in order to closely match the target roll periods of the prototype vessels.

The model yachts were moored in the model using a Mediterranean style mooring, with two bow lines and two stern lines. The stern lines were short and did not have any springs, since these lines are short enough in reality that their stretch is limited. The bow lines were longer and fitted with light-duty springs to simulate the elasticity of real mooring lines and maintain some pre-tension in the lines (see Figure 16).

Although the yachts and the moorings were not modelled in a rigorous manner, and no quantitative measurements of ship motions or mooring loads were obtained, useful qualitative information was obtained that supported and confirmed the assessment of safe agitation levels based on wave conditions alone.

Figure 16. Moored yacht in the model marina.

EFFECTS ON SURROUNDING FACILITIES

The possible effect of some of the refined Pierhead marina layouts on the adjacent existing Fishing Harbour and Careenage area was specifically investigated. These investigations focused on the effect (if any) of the proposed marina on agitation in these areas, as well as stability of the main Fishing Harbour breakwater structure.

The vertical breakwater structure of the proposed marina was found to reflect some incident wave energy towards the Fishing Harbour. This in turn slightly increased the wave heights approaching the existing facility and moderately increased the wave height within the mooring basin (inner harbour) for all conditions up to the 5-year wave event. Several alterations to the Fishing Harbour layout were then tested, representing a range of costs and levels of construction effort, to investigate the effects on agitation in the existing basin. It was concluded that a simple modification involving extending the inner North Breakwater would mitigate the effects from the proposed Pierhead marina.

Stability of the main Fishing Harbour breakwater was analyzed by incrementally augmenting the wave conditions to the point of imminent failure of the structures. This process was repeated for the case with, and without, the proposed Pierhead breakwater in place. It was found that there was marginally more damage to the Fishing Harbour breakwaters with the proposed marina breakwater installed. However, a comparison of the characteristic wave heights throughout the physical model basin showed that some of the additional wave height could be attributed to increased incident waves. The presence of the proposed Pierhead breakwater resulted in reflections back to the wave generator and slightly higher waves in the enclosed physical model basin. Therefore, these additional investigations were considered conservative, and it was concluded that the apparent mild increase in damage shown in the model was not discernibly different from the existing conditions.

The possible effect of deepening the entrance to the Careenage was also considered, both with and without the proposed marina structures in place. Results indicated that the introduction of the Pierhead marina would produce no net gain in wave height/energy within the Careenage, as long as the (inner) marina fuel pier revetment was constructed. Furthermore, if the Careenage entrance was to be dredged deeper, it is not expected that there would be any noteworthy increase in wave height at the inner Careenage under most conditions.

CONCLUSIONS

Three dimensional physical model studies at 1:50 scale have been conducted to support the design of a new mega-yacht marina proposed for a site in Carlisle Bay near the city of Bridgetown, Barbados. This scale was selected to make optimal use of the testing facility, minimize scale effects, and provide reliable, trustworthy results. The physical model was used to investigate many key issues including: wave agitation within the marina, wave overtopping, wave impact forces, the stability of revetments and scour protection stone, and the behaviour of moored pleasure yachts. The model was used to assess the performance of the new marina in a range of operational and extreme metocean conditions, to understand how the various design parameters affect the performance of the marina, and to refine and verify preliminary designs.

It was determined that the length of the breakwater could be reduced by 40-50 metres from the length that was initially proposed, and the subsequent level of agitation within the marina would remain at an acceptable level. The crest elevation of the breakwater should be set in the range of +4.0 metres, with local increases and decreases in areas with greater or lesser risk levels. The addition of a revetment near the marina entrance is effective enough that it had a similar effect to extending the breakwater by 10 metres (therefore allowing a shorter breakwater). A revetment at the inner marina reduces the local wave heights where floating docks and smaller vessels may require greater protection. Wave pressures on the breakwater need to be averaged over a large area to produce appropriate design loads on the structure.

It is not expected that the installation of the proposed marina would impact the surrounding area significantly, in particular nearby boat mooring facilities. The knowledge and results gained from the physical model study have been used to support the final design of the new marina facility.

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